

AN IMPROVED MEANS OF REINFORCING ADOBE WALLS - EXTERNAL VERTICAL REINFORCEMENT

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ABSTRACT

Traditional adobe (mudbrick) houses are highly susceptible to damage and destruction during seismic events. This vulnerability is particularly acute in developing countries where traditional construction practices and resource limitations result in large stocks of at-risk houses. The current favoured method to improve the earthquake resistance of new adobe houses includes the use of pilasters / buttresses, a crown/ring beam, internal vertical reinforcement (e.g. bamboo, cane, plastic tubing) and internal horizontal reinforcement (e.g. bamboo, cane, wire mesh). Although this method has been observed to significantly delay structural collapse, this paper raises some questions about the capacity of this form of reinforcement to delay the onset of cracking at low intensity ground motions. Furthermore, the complexity of the system is a major obstacle to its widespread use.

An alternative system is being developed and tested at the University of Technology, Sydney (UTS), Australia. This system retains the use of a crown/ring beam and internal horizontal reinforcement (wire mesh), however the vertical reinforcement (bamboo) is placed externally and attached to the wall after construction. The bamboo is securely tied to the horizontal and vertical reinforcement and the ring beam, creating a stable matrix. This system can also be slightly modified for the seismic retrofitting / strengthening of existing dwellings. Shake table testing at UTS has shown the system to be an effective means of impeding initial cracking, as well as delaying major structural damage and ultimate collapse. Scale model (1:2) u-shaped wall panels are being subjected to transient dynamic loading using a shake table to evaluate the response to out-of-plane seismic forces. Time-scaled input spectra are being used to induce damaging resonance conditions and the force-displacement characteristics and failure mechanisms of different reinforcement systems are being studied to determine their resistance capacity. This paper presents the results of shake table tests at UTS, as well as providing some discussion on the opportunities and challenges for implementation.

INTRODUCTION

One of the vital tasks for civil engineers is to minimise life loss, injury and property damage under extreme dynamic loading. Devastating earthquakes in Asia, The Middle East, Africa and Latin America have served as recent reminders of the vulnerability of non-engineered, low-cost dwellings to seismic forces. The loss of life and livelihood is often drastic, with millions of people in the poorest communities most severely affected (Figure 1). Adobe (mudbrick) housing is particularly vulnerable because of its inherently brittle nature, wide-spread use, generally poor construction quality and the limited awareness of concepts of aseismic design and construction. Despite this limitation, there is little doubt that adobe will continue to be the choice construction material for the majority of the rural poor who simply cannot afford any alternative.

Adobe mudbrick research at the University of Technology, Sydney (UTS) is focussed on developing and assessing methods to reduce the vulnerability of adobe housing to the effects of extreme dynamic loading, such as earthquakes. This research combines traditional building

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techniques, inexpensive reinforcement systems and state-of-the-art facilities, including the UTS shake table, to produce low-cost, low-tech solutions for application in developing countries.



Figure 1. Earthquake impact in Iran, February 22, 2005. The Mw 6.4 earthquake destroyed thousands of adobe homes and caused over 500 fatalities (Homavandi /Reuters).

CURRENT APPROACH

The current favoured method to improve the earthquake resistance of new adobe houses includes the use of pilasters / buttresses, a crown/ring beam, internal vertical reinforcement (e.g. bamboo, cane, plastic tubing) and internal horizontal reinforcement (e.g. bamboo, cane, wire mesh). This method, or derivations thereof, have been recommended by a variety of sources (e.g. IAEE, 1986; RESESCO, 1997; Equipo Maíz; 2001; Pérez, 2001; Dowling, 2004a; PUCP *et al*, no date). Furthermore, this method has been used for improved adobe training and construction projects around the world, and in particular in El Salvador since the earthquakes of 2001 (Dowling, 2004b). This paper raises some questions about the capacity of this form of reinforcement to delay the onset of cracking at low intensity ground motions. Furthermore, from a practical viewpoint, the use of internal vertical reinforcement is fraught with difficulties. The method is complex and time consuming, and as a result, it is unlikely to be widely accepted in practice, particularly without the support of trained and skilled masons (who don't like the system anyway!). Complications arise in each stage of wall construction, from preparation of the foundation and initial alignment of reinforcement, to adjusting and trimming the bricks to fit the reinforcement (natural products such as bamboo are seldom straight and defect-free), to the placement and adequate connection of the ring beam. In times of wet-weather during construction, the walls are difficult and time-consuming to completely cover and protect. There are also some concerns about the durability of the natural materials commonly used as internal vertical reinforcement (e.g. bamboo, reeds, timber). There is little doubt that when the internal reinforcement is completely encased it is afforded some protection from attack by insects, air and moisture, however, it is extremely difficult to adequately assess the condition of the reinforcement over time, and it is impossible to change the reinforcement if deterioration has occurred.

These reasons have been the major motivation to explore alternative options, which provide significantly improved seismic performance, yet are still simple, affordable and easily repairable if deterioration does occur.

The predominant failure modes of traditional unreinforced mudbrick houses subjected to earthquake loads are vertical cracking at the intersection of orthogonal walls, and vertical mid-

span cracking due to out-of-plane flexure (Zegarra *et al.*, 1997; Flores, *et al.*, 2001; Dowling, 2004a). Improvement systems or techniques which are designed to reduce damage and destruction of adobe structures should primarily address these main failure modes. In order to assess the capacity of different improvement systems to reduce such failure a series of shake table tests of 1:2 scale u-shaped adobe mudbrick wall units are being undertaken at UTS.

DESCRIPTION OF SPECIMENS

General

The specimens tested were u-shaped adobe wall units, in 1:2 scale, as shown in Figure 2. Specimen dimensions and configuration satisfy design criteria recommended in relevant guidelines (e.g. IAEE, 1986; RESESCO, 1997). Each specimen consisted of 333 full bricks (150 x 150 x 50 mm) and 56 half bricks (150 x 70 x 50 mm) laid in stretcher bond. Bricks were dipped in water prior to laying. The mortar was made of the same mud material as the bricks; all mortar joints were 12 - 13 mm thick. A downward restraining force was applied to the tops of the 'wing' walls (acting as in-plane shear walls) to simulate the restraint provided by a continuous wall, and to prevent over-turning of the complete unit. This effectively transfers the bulk of the seismic loading to the areas of main interest: the vulnerable out-of-plane long wall, and the corner connections. The restraining force was applied by tension bars between timber platens and beam resting on the walls, and the concrete base. A pressure of ~125 kPa was applied, representing 50% of the load which induced initial cracking in a series of compression tests on mudbrick masonry prisms. This applied restraining force is a significant difference between this experiment and other dynamic tests on u-shaped adobe wall panels, which do not include any 'wing' wall restraint (e.g. Zegarra *et al.*, 1997) and thus neglect the contribution of the shear walls in the dynamic response of the system.

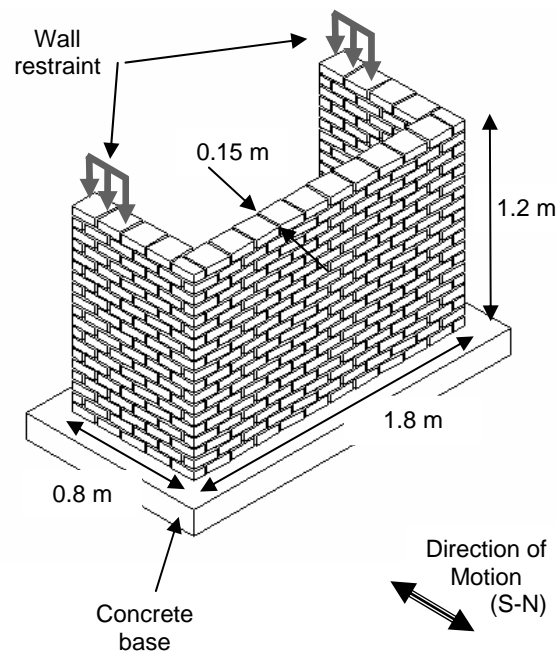


Figure 2. Specimen configuration and dimensions.

The specifications of each wall unit are summarised in Table 1. Additional details are provided below.

Table 1. Specifications of u-shaped wall units

Specimen	System
3A	Unreinforced, traditional (Figure 3)
3B	Corner pilasters / buttresses only (Figure 4)
3E	Internal horizontal chicken wire mesh (every three courses) External vertical bamboo (inside and outside) Timber ring beam (Figure 5)
3G	Internal horizontal chicken wire mesh (every three courses) Internal vertical bamboo Timber ring beam (Figure 6)
3I	Internal horizontal chicken wire mesh (every three courses) External vertical bamboo (inside) External horizontal bamboo (outside) Timber ring beam (Figure 7)

Specimen 3A

Specimen 3A was built to represent an unreinforced, traditional form of construction (Figure 3).

Specimen 3B

The only alteration in Specimen 3B was the addition of corner pilasters / buttresses (Figure 4). The pilasters were fully integrated with the wall via stretcher bond, with pilaster length = width of wall + one mortar joint (ie. ~162 mm representing ~325 mm in full-size).

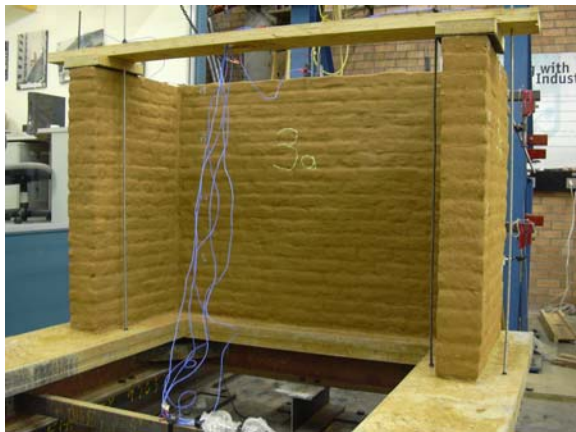


Figure 3. Specimen 3A ready for testing.



Figure 4. Specimen 3B ready for testing.

Specimen 3E

Specimen 3E was reinforced with external vertical bamboo reinforcement, internal horizontal chicken wire mesh and a timber ring beam (Figure 5a). Polypropylene strings were weaved through the horizontal chicken wire mesh (perpendicular to the wall). The mesh was then laid in the mortar joints every three courses during construction. 2 mm wire loops to tie down the

timber ring beam were laid in the mortar joint four courses below the top of the wall, at 150 mm spacing. After construction and curing of the wall the vertical bamboo was firmly connected to the concrete support base and attached to the wall (via the polypropylene string). The timber ring beam was connected to the top course of adobe bricks via pins (resisting shear forces) and the 2mm wire loops (resisting uplifting forces), as well as securely attached to the external bamboo by 2mm wire and staples (Figure 5b). The resulting structure was an integrated matrix, with strong connection between the foundation, structure and reinforcement.



Figure 5a. Specimen 3E ready for testing. Note external vertical bamboo (inside and outside).

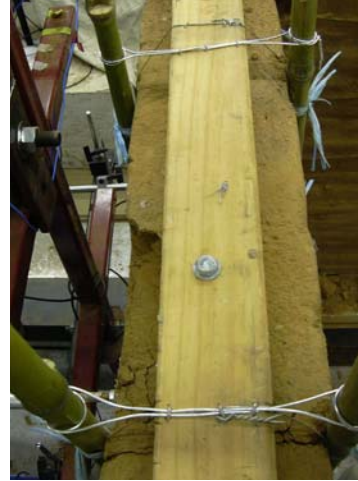


Figure 5b. Connection of ring beam, wall and bamboo (Specimen 3E).

Specimen 3G

Specimen 3G was reinforced with internal vertical bamboo reinforcement, internal horizontal chicken wire mesh (every three courses), and a timber ring beam. The bamboo was securely attached to the foundation prior to construction, and half bricks and full bricks with notches were configured to encase the bamboo at alternate courses (Figure 6).

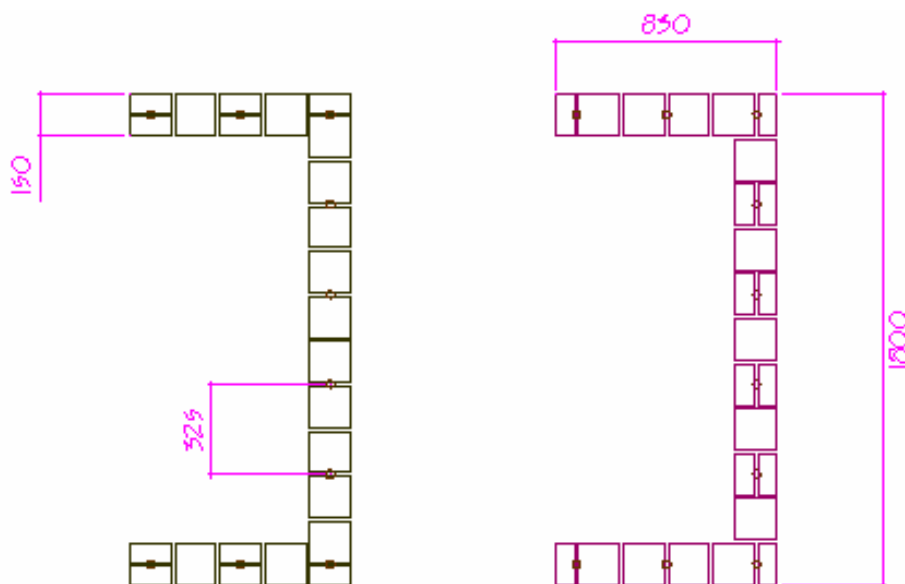


Figure 6. Plan layout for Specimen 3G

Specimen 3I

Specimen 3I was constructed in a similar manner to specimen 3E, except that on the outside face of the structure, horizontal bamboo reinforcement was used instead of vertical bamboo reinforcement (Figure 7a). The horizontal bamboo was securely connected at the corners by drilling the bamboo and inserting small wire pins (Figure 7b).



Figure 7a. Specimen 3I ready for testing. Note external horizontal bamboo (outside) and external vertical bamboo (inside).



Figure 7b. Connection of external horizontal bamboo at corner (Specimen 3I).

DESCRIPTION OF EQUIPMENT

Each wall unit and support frame was instrumented with accelerometers and LVDT displacement sensors to record the relative movement and dynamic response of the structure, with particular focus on the out-of-plane, 'long' wall. Data for these sensors was recorded on both a PC-based digital data acquisition system, and two analogue Yokogawa dynamic signal analysing recorders. The dynamic simulation was done on the state-of-the-art MTS uni-axial shake table located at the University of Technology, Sydney. The table specifications are shown in Table 2.

Table 2. UTS shake table specifications

Size of table	3m x 3 m
Maximum Payload	10 tonnes
Overturning Moment	100 kN-m
Maximum Displacement	±100 mm
Maximum Velocity	±550 mm/sec
Maximum Acceleration	±2.5g or 0.9g (full load)
Testing Frequency	0.1 – 50 Hz

DESCRIPTION OF INPUT TIME HISTORY FOR SHAKE TABLE SIMULATIONS

The approach taken to choose, modify and apply the input time history has been described in detail in Samali, *et al.* (2005). A summary of this process is included below:

Proposed design spectrum

The proposed design spectrum is a modified version of the El Salvador design spectrum (Figure 8) reported by López *et al.* (2004). Modifications were made to better envelope recent and more severe earthquakes (elevation of peak spectral acceleration level to 1.5g) and to cover a wider range of frequency contents (extension of the flat portion of the spectrum to a period of 0 seconds), which is considered to be a conservative approach for very stiff structures (such as adobe and other masonry). Since adobe structures generally exhibit an elastic response followed by a brittle failure mechanism, the Elastic Response Spectra was used in this study.

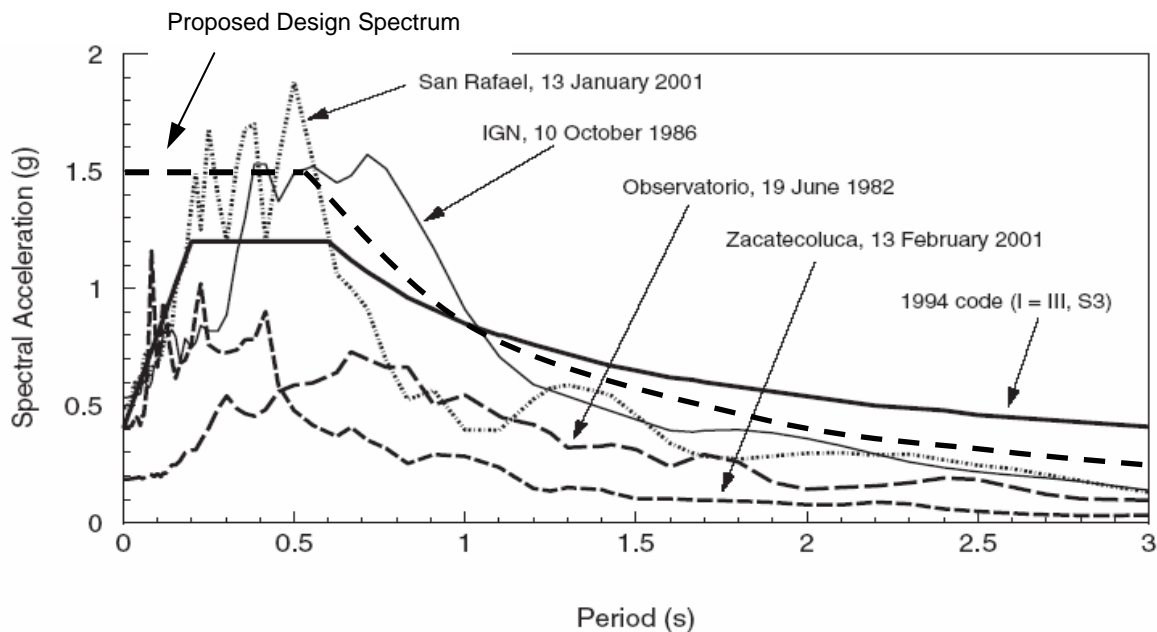


Figure 8. Proposed Design Spectrum (thick dashed line) vs Elastic Response Spectra developed from some El Salvador earthquakes (courtesy of Lopez *et al.* 2004)

Selection and verification of input time histories

In this study the input time history from the January 13, 2001 El Salvador earthquake (M_w 7.7) was used (Figure 9). (This earthquake, in combination with a M_w 6.6 earthquake on February 13, 2001 in the same area, caused the destruction of over 110,000 adobe houses (DIGESTYC, 2001; Dowling, 2004b)). The Test Response Spectrum (TRS) for the January 13, 2001 earthquake was plotted against the proposed design spectrum (Figure 10a), and found to envelope the portion of the proposed design spectrum (above the design acceleration of 1.5g) in the high frequency resonant range (6.5 -16 Hz) which is typical of adobe masonry. Figure 10b shows the Test Response Spectrum (TRS) in relation to the proposed design spectrum in this dominant frequency range. In order to assess the performance of the mudbrick specimens with respect to the proposed design spectrum (1.5g in the bandwidth of interest) a ‘target frequency zone’ (shaded circle) was identified.

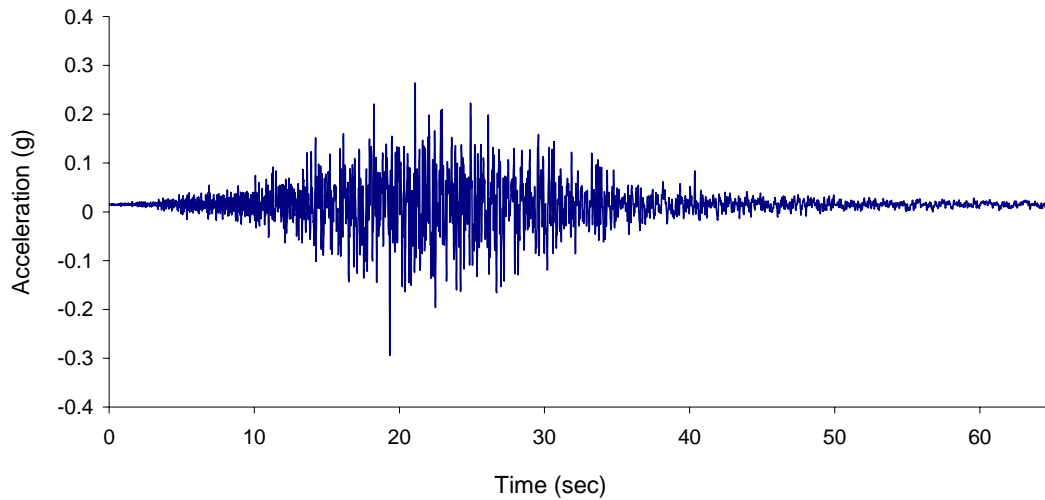


Figure 9. Unscaled input time history (full), January 13, 2001 El Salvador earthquake (UCA Station, Zacatecoluca).

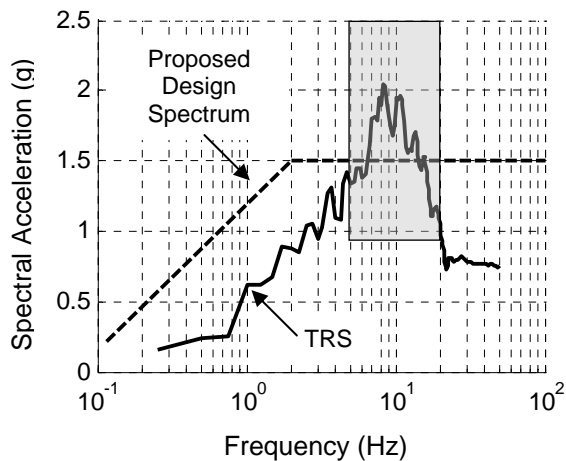


Figure 10a. Calculated Test Response Spectrum (TRS) (solid line) in relation to the proposed design spectrum (dashed line). The shaded section is shown in Fig. 10b.

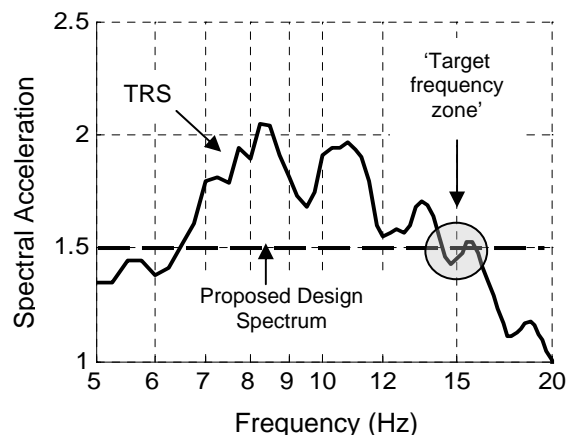


Figure 10b. Test Response Spectrum (TRS) (solid line) in relation to the target maximum acceleration (1.5g)

Scaling of input time histories

In order to induce damaging resonance conditions and maintain dynamic similitude between different specimens, the frequency ratio (defined as the ratio of the dominant frequency of the input excitation to the dominant first natural frequency of the structure) must be similar. Clearly it is not possible to alter the natural frequency of each specimen, so scaling of the input spectra is undertaken. This can be done by adjusting the input spectra such that the natural frequency of the specimen lies within the adjusted 'target frequency zone'. By means of modal analysis, the first resonant frequency (natural frequency) of each specimen was identified via the Frequency Response Function (FRF) calculations (Figure 11). The relevant input spectra time scaling factor was then calculated for each specimen (Table 3).



Figure 11. Frequency Response Function (FRF) of specimen 3A from the modal hammer tests.

Table 3. Specimen first natural frequency, calculations and time scaling factors.

Specimen	First Natural Frequency (Hz)	Calculation	Time Scaling Factor
3A	29.6 Hz	29.6 / 15.0	1.97
3B	34.1 Hz	34.1 / 15.0	2.27
3E	30.8 Hz	30.8 / 15.0	2.05
3G	24.8 Hz	24.8 / 15.0	1.65
3I	31.6 Hz	31.6 / 15.0	2.11

Scaling of intensity

In addition to the importance of considering frequency relationships, the other main factor contributing to damage of structures due to seismic loading is wall displacement, which is directly affected by the ground (shake table) displacement. The shake table displacement can be scaled to produce ground motion simulations of varying intensities. In order to study the behaviour and performance of the structures at different load levels a series of simulations was undertaken with varying displacement intensities, ranging from 40% - 200% for the unscaled (time) input and 20% - 125% for the time scaled input.

RESULTS

General results

Table 4 shows the observations at each testing stage. The first three simulations used the unscaled (time) input (Figure 9), with increasing displacement intensity: 40%, 100% and 200%. In each case (except 3G) this resulted in no damage to the structures (even at twice the original intensity) and confirms the necessity to use time-scaled input spectra to induce damaging resonance conditions.

Table 4. Results for Specimens 3A, 3B, 3D, 3E, 3G and 3I

Time	Simulation	Displacement Intensity	3A	3B	3E	3G	3I
Unscaled							
	S1	40%	No damage	No damage	No damage	No damage	No damage
	S2	100%	No damage	No damage	No damage	No damage	No damage
	S3	200%	No damage	No damage	No damage	Significant cracking	No damage
Scaled							
	S4	20%	No damage	-	No damage	-	No damage
	S5	50%	No damage	No damage	No damage	-	No damage
	S6	75%	Severely damaged (collapse imminent)	-	No damage	-	No damage
	S7	100%	-	Severely damaged (collapse imminent)	Minor hairline cracking	-	Very minor hairline cracking
	S8	125%	-	-	Severely damaged but collapse NOT imminent	-	Minor cracking (<3mm). Still structurally sound
	S9, S10	75% (x2)	-	-	-	-	Progressive additional damage
	S11-S14	100% (x4)	-	-	-	-	Progressive additional damage leading to severe damage, but collapse NOT imminent

Specimen 3A

Specimen 3A exhibited distinct and classic failure patterns (Figure 12), indicative of damage to real houses subjected to real earthquakes. A number of factors contributed to the failure of the specimen, including:

- i) Flexure induced in the out-of-plane 'long' wall, causing a splitting-crushing cycle at the mid-span of the wall and the intersection with the orthogonal shear 'wing' walls. (Figure 13 shows the flexural displacement in the 'long' wall prior to formation of the first significant cracks in the specimen.)
- ii) Large relative displacement between the 'flexible' out-of-plane 'long' wall and the stiff in-plane shear 'wing' wall, leading to tearing failure of both the mortar-brick interface and the individual brick units at the corner intersection (vertical corner cracking).
- iii) Overturning of the 'long' wall, which was induced by the lack of connection with the 'wing' walls, a result of the aforementioned vertical corner cracking.



Figure 12a. Specimen 3A after simulation S6



Figure 12b. Specimen 3A after simulation S6

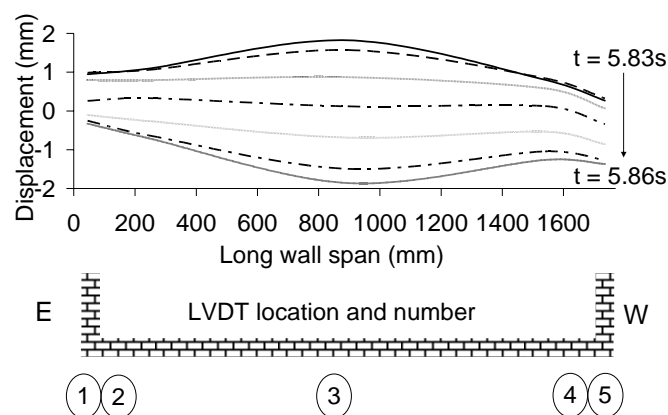


Figure 13. Specimen 3A: flexural displacement of 'long' wall from $t = 5.83s$ to $t = 5.86s$ (prior to initial significant cracking in simulation S6)

Specimen 3B

Specimen 3B displayed similar failure patterns to Specimen 3A, although the vertical mid-span cracking in Specimen 3B was more pronounced (Figure 14a). A surprising result was the location of the vertical crack (Figure 14b) at the NW intersection of the walls (left side of Figure 14a). It was anticipated that the pilaster would provide some restraint to the overturning moment which induces tearing failure in the wall, and that cracking would initiate on the inside corner, due to the build-up of shear and bending stresses (as seen in the NE corner opposite). This result indicates that the corner pilaster in itself may not be providing the stability desired.

In addition to uncertainties relating to the aseismic contribution of the corner pilasters, there are a number of practical reasons to question their use, including the increased labour, complexity and cost required for the preparation and construction of the foundation, plinth, pilaster and roof systems. This aspect should be the subject of further research, with the optimal pilaster dimensions (depth and height) assessed and overall performance compared with other improvement systems. Although not tested in this research program at UTS, the use of intermediate (mid-span) pilasters in long walls are still advocated by the authors of this paper, and should be the subject of future research.

An error in the simulation procedure meant that the specimen was not subjected to the 75% intensity time-scaled simulation. Major cracking occurred during the 100% intensity time-scaled simulation, although it is expected that minor cracking may have occurred during the 75% intensity simulation. In any case, the failure patterns are considered to be accurate, and cracking and failure of the structure can be judged at occurring between 75 and 100% intensity.



Figure 14a. Specimen 3B after simulation S7.



Figure 14b. Specimen 3B after simulation S7 (NW corner).

Specimen 3E

Specimen 3E performed significantly better than the previous specimens, and withstood the forces of a 125% intensity time-scaled simulation without collapse (Figure 15). Collapse of the structure was prevented by the combined contributions of the bamboo, chicken wire mesh and ring beam. This integrated matrix acted to restrain movement, and absorb, dissipate and redistribute energy within the structure. The main factors contributing to damage discussed in Specimen 3A (above) were still evident in the reinforced specimen, however, the impacts were significantly diminished:

- i) Flexure in the out-of-plane 'long' wall was reduced, although vertical mid-span cracking was still caused (obscured by bamboo in Figure 15a), as well as diagonal cracking due to out-of-plane 'bulging' of the 'long' wall.
- ii) The relative displacement between the 'flexible' out-of-plane 'long' wall and the stiff in-plane shear 'wing' wall was reduced, although vertical corner cracking (Figure 15b) was still caused during the high intensity simulations. It is expected that if the ring beam were continuous along the shear 'wing' wall then the development of the vertical corner cracking would have been further delayed.
- iii) After the development of vertical cracking in both corners, the 'long' wall behaved as a semi-rigid single body (panel) experiencing an overturning moment. The panel was still restrained by the ring beam, chicken wire and bamboo, which prevented complete overturning. Figure 16 shows the large displacement of the top / mid-span of the 'long' wall leading to significant damage during simulation S8.

In practice, an attractive finish could be easily applied by covering the wall and reinforcement (e.g. bamboo) with an appropriate render (e.g. lime and sand). If the reinforcement was too thick, it could be split, and/or a small vertical channel could be carved into the wall in which to bed the reinforcement. Periodically, the render could be removed at certain locations and the condition of the reinforcement assessed. Deteriorated reinforcement could be easily removed and replaced, and a new render applied. It is expected that the render may crack and spall during future seismic events, but more importantly, the structure itself will be significantly safer than an unreinforced structure.



Figure 15a. Specimen 3E after simulation S8



Figure 15b. Specimen 3E after simulation S8

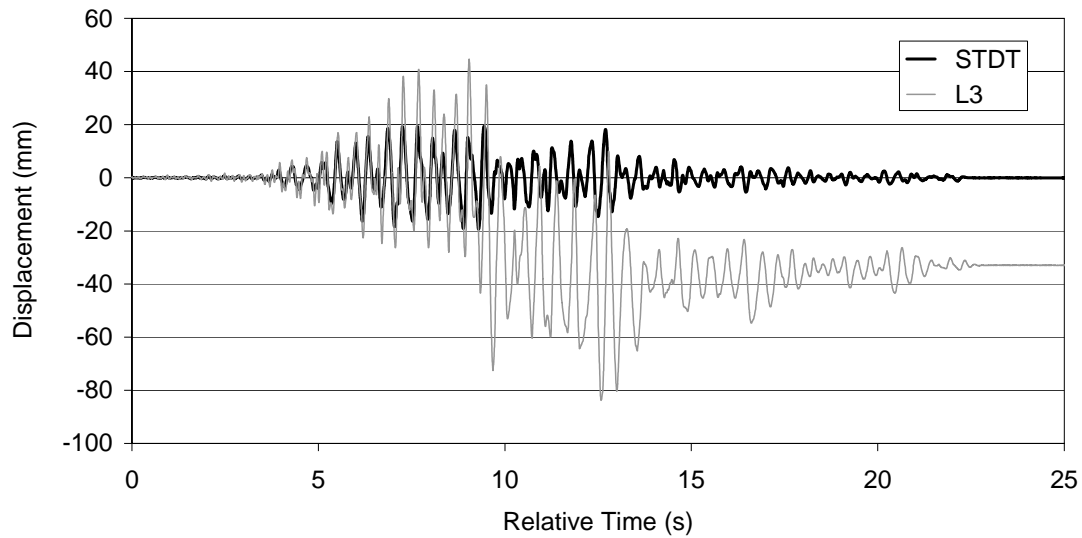


Figure 16. Displacement of shake table (STDT) and top / mid-span of 'long' wall (L3) for specimen 3E during time-scaled simulation S8 (125% intensity)

Specimen 3G

The behaviour of Specimen 3G was a great surprise: significant cracking occurred after the 200% intensity unscaled (time) simulation (Figure 17). This situation did not occur in any of the other specimens, and suggests significant weaknesses in the structure. The mid-span cracking in the 'long' wall occurred at the location of the internal vertical bamboo, indicating a significant difference in the response of the stiff bricks and the flexible bamboo (this may also account for the lower global stiffness of the structure prior to testing, as shown in Table 3). Results indicate that the internal vertical bamboo has introduced discontinuity to the structure, which has created planes of weakness. The results raise questions about the structural improvement which can be expected by using internal vertical reinforcement, and should be the focus of further testing.



Figure 17a. Specimen 3G after simulation S3 (Arrows indicate the location of internal vertical bamboo)



Figure 17b. Specimen 3G after simulation S3

It is proposed that the 1:2 linear scaling of the specimen may have contributed to the poor performance of the structure. The use of half bricks with notches to accommodate the bamboo resulted in an effective cover (distance from face of wall to bamboo) of ~60 mm (even less when the bamboo is misaligned), whereas in full-size (1:1) application, the effective cover is in the region of ~120-130 mm for a 300 mm thick wall. This scaling factor may adversely affect the results in this particular case. It should also be noted that the specimen was not tested beyond the 200% unscaled (time) simulation. Retrospect suggests value in testing specimens to complete destruction in order to assess the capacity of the structure to resist total collapse. This aspect has been noted and subsequent specimens (e.g. Specimen 3I) have been subjected to additional simulations, beyond the occurrence of significant cracking.

In order to further evaluate the performance of adobe walls with internal vertical reinforcement, an additional specimen has been constructed (in the same manner as 3G) and will be tested prior to SismoAdobe2005.

Specimen 3I

Specimen 3I performed in a similar manner to Specimen 3E, although the initiation and severity of cracking appears to have been delayed and reduced in Specimen 3I. The specimen maintained excellent structural integrity, even during high intensity simulations (100% and 125%). After being subjected to the 125% intensity time-scaled simulation (Figures 18 and 19), the specimen was then subjected to two time-scaled simulations at 75% intensity, followed by four time-scaled simulations at 100% intensity. The results after this series of severe earthquakes are shown in Figures 20 and 21 and reveal the structure to be still safely standing, despite being significantly damaged.

The main practical shortcoming of this method is the large cross-over of the horizontal bamboo reinforcement at the corners (this would also be complicated at window and door openings). This cross-over would be impractical if a render is to be applied to the wall, however, if the wall is to be left un-rendered, then the use of external horizontal reinforcement is highly recommended.



Figure 18a. Specimen 3I after simulation S8



Figure 18b. Specimen 3I after simulation S8

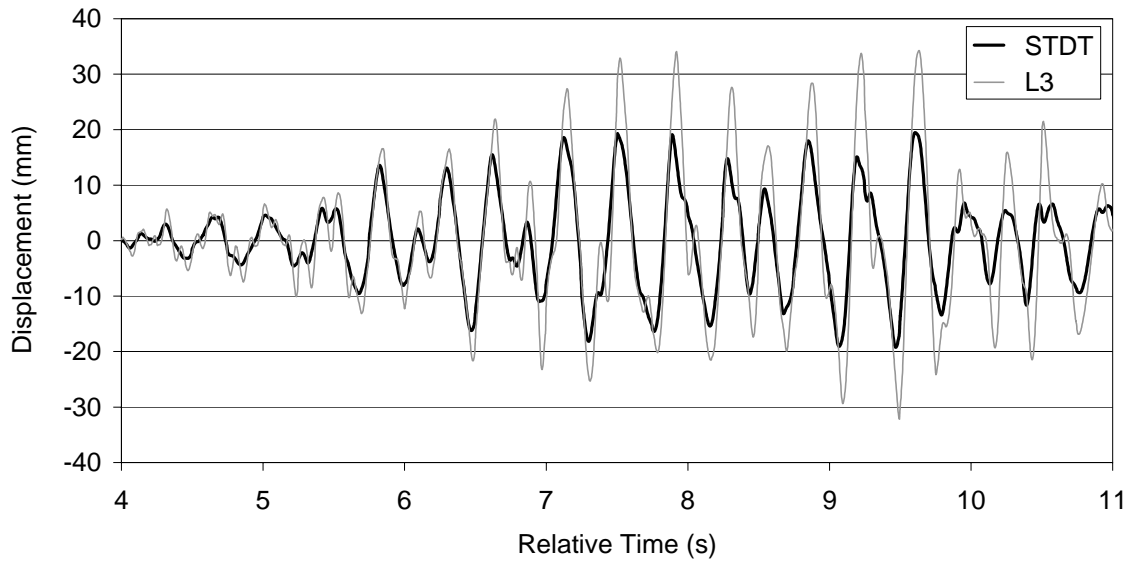


Figure 19. Displacement of shake table (STDT) and top / mid-span of 'long' wall (L3) for specimen 3I during the period of intense shaking leading to cracking during time-scaled simulation S8 (125% intensity)



Figure 20. Specimen 3I after simulation S14



Figure 21a. Specimen 3I after simulation S14



Figure 21b. Specimen 3I after simulation S14

CONCLUSIONS AND RECOMMENDATIONS

Test results reveal that the use of u-shaped adobe wall panels, with appropriate ‘wing’ wall constraint, exhibit classic failure patterns when subjected to a suitable input time history. Damages were consistent with real structures subjected to real earthquakes.

Test results confirm the importance of appropriate time scaling of input time history to induce damaging resonance conditions in a structure. Time scaling is also necessary to ensure dynamic similitude between specimens, such that accurate comparisons may be made between the performance of different specimens.

Test results challenge the assumption that corner pilasters / buttresses will adequately restrain the out-of-plane overturning moment induced in a wall, which create vertical corner cracking due to tearing failure in the orthogonal wall.

Test results indicate that there is some uncertainty relating to the structural performance of internal vertical reinforcement, with significant cracking occurring at a lower intensity than expected. This aspect, coupled with the complexity of construction, raises questions about the viability of using this form of reinforcement.

Test results indicate that significant improvement in the earthquake resistance of adobe mudbrick structures can be obtained by using external vertical and/or horizontal bamboo reinforcement, internal horizontal chicken wire mesh reinforcement and a ring beam. These additions, when securely tied together, create an integrated matrix which restrain movement, and absorb, dissipate and redistribute energy within the structure. The alterations proposed and tested in this paper are relatively simple and easy to undertake, and utilise low-cost and readily-available materials, making them appropriate for application in developing countries.

Further research will focus on modifications to the proposed system, including the effect of:

- Wall thickness
- Reinforcement location (spacing of bamboo)
- Ring beam configuration (extended further along shear wall, diagonal bracing)
- Openings (window and doors)
- Retrofit system (for existing buildings)
- Roof diaphragm.

Further computer analysis will be undertaken, considering the behaviour of the structure as recorded by the LCDT displacement sensors and accelerometers.

These aspects are the focus of ongoing research at the University of Technology, Sydney (UTS), which includes the testing of a total of eleven u-shaped wall panels and will culminate in the construction and testing of 1:2 scale complete houses, representing traditional and improved construction systems.

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